On advanced numerical techniques for the modeling of bolt reinforced rock mass

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ABSTRACT: Rock bolting plays an important role in different geo-engineering applications and its numerical modelling is crucial for the analysis and design of rock structures. Continuum modelling simulation of bolt-reinforced rock masses requires specific techniques to properly model the reinforcement system and its interaction with the rock mass, which often exhibits a nonlinear softening/brittle response. In this context, strain localization might occur, which, in turn, may affect numerical convergence and the quality of results. This paper presents some advanced numerical techniques implemented in PLAXIS to overcome the abovementioned challenges. Firstly, a regularization technique is implemented for an extended version of the Hoek-Brown failure criterion with strain softening. Secondly, the formulation of the structural bolt element interacting with the rock mass is developed. Finally, the robustness and accuracy of these techniques are discussed via a numerical example of a typical underground mining excavation problem.

Keywords: Numerical modelling, Hoek-Brown with Softening, Rock reinforcement, Underground excavations, Viscous regularization.

1 INTRODUCTION

Rock bolting has been widely used in rock engineering practice. This technology aims at preserving and improving the overall rock mass properties through a load transfer mechanism between the rock and the reinforcement system. Therefore, numerical modelling of bolt-reinforced rock mass is crucial for the analysis and design of rock structures.

The continuum modelling approach, notably the Finite Element (FE) method, has proven to be a powerful tool for large-scale applications as it provides relatively high accuracy with reasonable computational cost, especially compared to discrete element approaches. When dealing with the simulation of bolt-reinforced rock masses, specific techniques are required to properly model the rock mass, the reinforcement system (rock bolts) as well as their interaction. Moreover, rocks often exhibit nonlinear brittle behaviour with softening response which may lead to very restrictive time-step constraints and, more seriously, significant mesh dependency and unreliable numerical results when strain localization occurs (Walton et al. 2014). These aspects may affect numerical convergence and the quality of results, especially in the context of rock-structure interaction. Thus, advanced techniques are needed to enhance computational robustness and reliability.

This paper presents some advanced numerical techniques implemented in the FE code PLAXIS (Bentley Systems, Inc., 2023) to overcome the aforementioned challenges. Concretely, a viscous regularization technique was implemented in the recently proposed Hoek-Brown with Softening (HBS) model (Marinelli et al. 2019). This technique allows for mitigating the mesh-dependency of the computed results due to strain localization. Moreover, a versatile structural element has been recently introduced in PLAXIS, which can be used to model active and passive rock reinforcement systems such as rock bolts, cable bolts and ground anchors. First, these features/techniques are briefly introduced, and then their robustness and accuracy are illustrated through the numerical modelling of a typical bolt-reinforced mine drift.

2 VISCOUS REGULARIZATION TECHNIQUE IMPLEMENTED IN HBS MODEL

The HBS model is a non-associated strain softening elastoplastic model which incorporates a softening rule into the generalized representation of the Hoek-Brown (HB) yield envelope (Jiang, 2017) to consider material degradation in the post-peak regime. Both strength decrease and nonlinear dilation evolution are represented through a hyperbolic decay of the material properties. Details of the model formulation can be found in Marinelli et al. (2019) and are not recalled here.

Progressive failure in rock masses is usually conditioned by strain localization phenomena that substantiate the development of narrow shear bands which may lead to the collapse of rock structures due to plastic mechanisms. The numerical simulation of these phenomena suffers from the inherent mesh-dependency of numerical results (Pijaudier-Cabot & Bazant 1987), occurring in both softening materials and non-associative plastic materials (structural softening). To avoid unreliable mesh dependency, regularization techniques, such as Cosserat continuum, non-local plasticity, gradient plasticity and viscous regularization, should be used (De Borst & Duretz 2020). These techniques usually introduce an internal length controlling the thickness of the shear bands. Among these approaches, viscous regularization presents some advantages due to its simplicity for numerical implementation and its applicability for both softening and perfectly plastic materials. This method was implemented in the HBS model. Specifically, a fictitious viscous behaviour was introduced via the viscoplasticity theory of Perzyna (1966), in which the visco-plastic strain rate is defined by:

$$\dot{\varepsilon}_{ij}^{vp} = \gamma \langle \Phi(f) \rangle \frac{\partial g}{\partial \sigma_{ij}} \qquad (1)$$

where γ is the fluidity and g is the plastic potential and σ_{ij} indicates the components of the stress tensor. The symbol $\langle \cdot \rangle$ represents the Macauley brackets and $\Phi(f) = f / \sigma_{ci}$ is the viscous nucleus expressed as a function of the yield surface f and the intact compressive strength parameter σ_{ci}.

In this context, plastic strains defined in the HBS model are replaced by the above viscoplastic strains. As shown by Marinelli et al. (2019), the viscoplastic solution approaches the rate-independent plastic solution as fluidity goes to infinity. The fluidity should be calibrated based on two conditions: it should be large enough so that the material behaviour reasonably approximates the elastoplastic solution and small enough so that the mesh dependency is regularized. The performance of this regularization technique has been proven by numerical analysis carried out with the software PLAXIS (Marinelli et al. 2019; Zalamea et al. 2020 and Cammarata et al. 2023). The effect of this technique is herein further investigated when combined with the structural cable bolt element for the simulation of a reinforced underground excavation in rock.

3 STRUCTURAL CABLE BOLT ELEMENT

The cable bolt is a structural element implemented in PLAXIS 2D for simulating the reinforcement of rock mass by mobilizing axial forces to increase the strength of the rock mass.

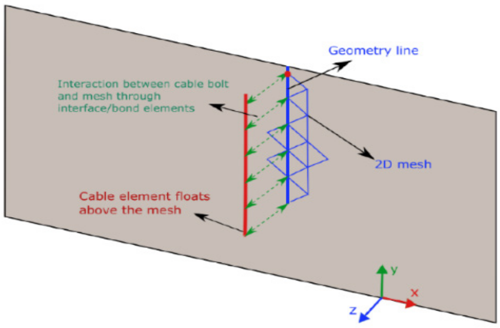


Figure 1. 2D FE conceptual representation of the cable bolt element.

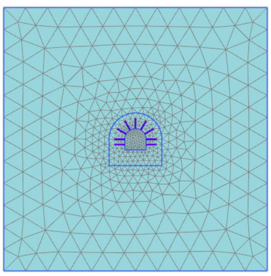
In 2D representation, the cable bolt is modelled by two components: a 3- or 5-noded 1D cable element, interacting with the finite element mesh (rock mass) by a zero thickness double-sided bond element (Figure 1). The former is characterized by an elastic-perfectly plastic material represented by its axial stiffness E and tension/compression limit $N_{p,tens/comp}$. The latter (also modelled by an elastic – perfectly plastic material) is characterized by its shear stiffness k_s, cohesive strength c_{bond} and friction angle φ_{bond}. In particular, the last two parameters provide the strength capacity of the bond (i.e., grout for grouted bolt, or friction for frictional bolt) as a stress-dependent function through the following relation:

$$T_{s,bond,max} = c_{bond} + \sigma_n \tan(\varphi_{bond}) \cdot Perimeter \qquad (2)$$

where σ_n is the confining stress normal to the cable axis while *Perimeter* refers to the exposed failure perimeter of the element (assuming that failure can occur either at the cable/grout or the grout/rock interface). It should be noted that the confining stress is calculated based on the current stress state in the rock mass surrounding the cable bolt.

4 MODELLING OF REINFORCED UNDERGROUND EXCAVATION

The underground excavation concerns a horseshoe-shaped drift 4.8 m wide and 4.8 m high within a homogeneous rock mass. The drift is located 350 m below the surface and subjected to a homogeneous and isotropic in-situ stress field of 10 MPa. To prevent the model boundaries from influencing the results, the domain is assumed 60 m wide and 60 m high. The mesh is refined in a zone surrounding the excavation, modelled by another horseshoe-shaped geometry. The drift excavation is reinforced with a set of 9 rows of grouted cable bolts along the boundary of the excavation, each row containing cables 2.4 m long spaced at 75 cm in the out-of plane direction. Figure 2 shows the model geometry, cable numbering and, as an example, one of the FE mesh used in the simulations. Note that in the 2D plane strain analysis hereafter, each cable represents a row and, for simplicity, the word ‘row’ is dropped.



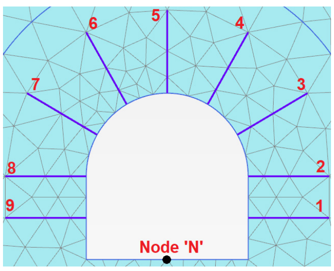


Figure 2. Geometry and example of FE mesh (left) and cable element numbers.

The drift is assumed to be mined in a good-quality layered sedimentary rock mass characterized by a GSI value of 80 and modelled by the HBS model. The residual strength complies with a 25%

reduction of the peak HB parameters combined with the rate of softening parameters guaranteeing a smoother peak-residual strength transition. The dilation parameters are consistent with a nonlinear decrease trend of the dilation angle from a peak value of 15° to a nil residual value. For the grouted cables, failure is assumed to take place at the cable/grout interface, so the failure *Perimeter* (Eq. 2) is calculated based on the cable diameter (equal to 25.4 mm). All the rock and cable parameters are summarized in Table 1.

Table 1. Parameters for the HBS constitutive model (left) and cable bolt elements (right).

HBS model parameters	Value	Cable bolt parameters	Value
Young's modulus (E)	5.6 GPa	Young's modulus (E)	98.6 GPa
Poisson's ratio (ν)	0.25	Compressive strength ($N_{p,comp}$)	100 kN
Intact rock strength (σ_{ci})	25 MPa	Tensile strength ($N_{p,tens}$)	550 kN
Tensile strength factor (α)	0.2	Shear stiffness (k_s)	15 GN/m^2
Geological Strength Index (GSI)	80	Cohesive strength (c_{bond})	800 kN/m
Intact rock HB constant (m_i)	8		
Disturbance factor (D)	0		
Residual HB constants (m_{br} / s_r)	2.937/0.08		
Rate of softening ($B_m = B_s$)	0.0001		
Peak/residual dilation ($m_{\psi i}$ / $m_{\psi r}$)	0.5/0		
Rate of dilation ($B_{\psi i}$)	0.01		
Fluidity (γ)	1 day^{-1}		

Following the considerations reported in Section 2, a sensitivity analysis has been performed concerning the fluidity parameter, assuming a fixed time interval of 1 day. To be consistent, this time interval was also used in all the finite element analyses hereafter. As depicted in Figure 3, a value γ equal to 1 day^{-1} provides a reasonably close match between viscoplastic and elastoplastic responses at the material point level and, as shown later in the analyses, it provides sufficient regularization effect on numerical results. This property value has thus been herein adopted in the numerical model. Higher values of γ would provide a slightly closer match with the elastoplastic solution at the material point level, but less efficiency in the regularization effect in the numerical computations.

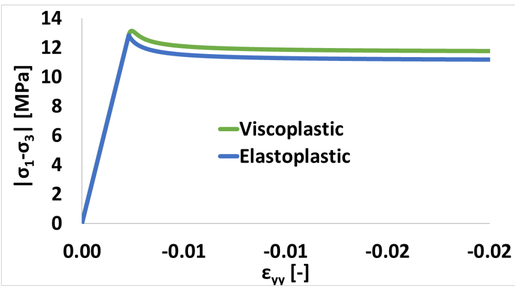


Figure 3. Stress-strain curve evolution from the peak to the corresponding residual stress and effect of the fluidity γ on the HBS model behaviour: elastoplastic versus regularized model for γ equal to 1 day^{-1}.

The mine drift excavation modelling considers the advancement of the face of the drift and the installation of reinforcements at a certain distance behind the face. In particular, it is assumed that the installation of the cable bolts takes place once 30% of confinement loss has occurred. To investigate the mesh-dependency, analyses have been evaluated by comparing the results for different meshes for both the elastoplastic and regularized viscoplastic solutions (using the calibrated value of fluidity previously introduced and the aforementioned time interval of 1 day).

Figure 4 shows the development of the shear bands along the drift boundary, after full excavation and for different representative meshes. As theoretically expected (De Borst & Duretz 2020), for the (non-regularized) elastoplastic solution, the size of shear bands, their formation and development are highly influenced by the mesh size (see the crown of the drift). On the other hand, for the viscoplastic solution, the shear bands at the corners of the invert become streamlined as the mesh is finer.

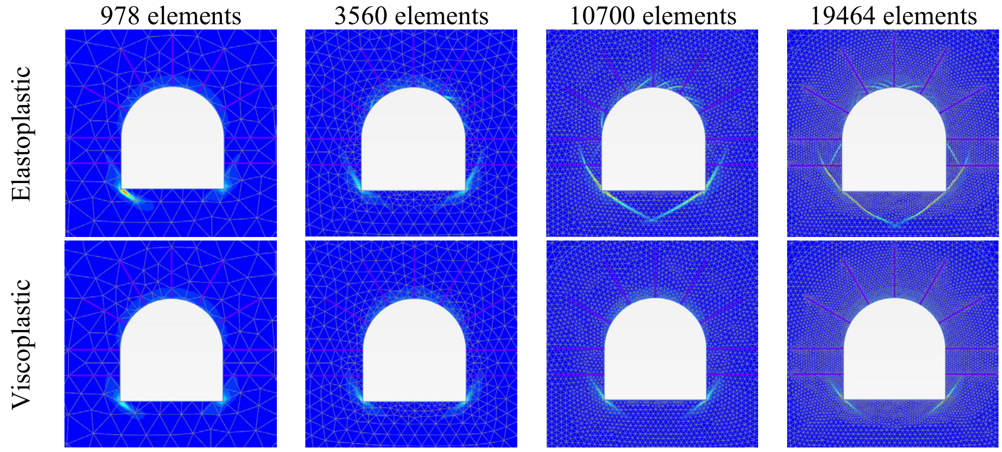


Figure 4. Shear band development and relative maximum deviatoric strain with varying mesh sizes. Note that the selected scale for deviatoric strains has been optimized to improve the quality of each image.

The evolution of the maximum deviatoric strain (localized in the shear bands) and displacements at node N (in the middle of the drift floor) with the number of finite elements is shown in Figure 5. Even if small differences are experienced in the magnitude of displacement between the two solutions, the viscoplastic analyses provide a more stable evolution trend. The viscoplastic analyses provide smaller but more stable values for the maximum deviatoric strain than those obtained by the elastoplastic solution.

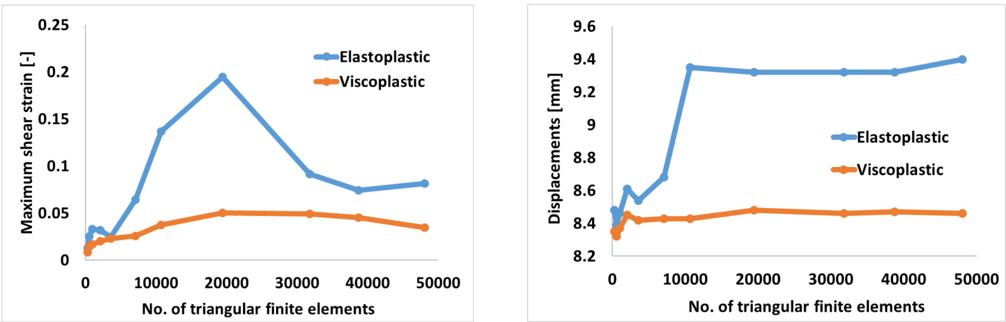


Figure 5. Maximum deviatoric strain and displacements at chosen node 'N' with varying mesh sizes.

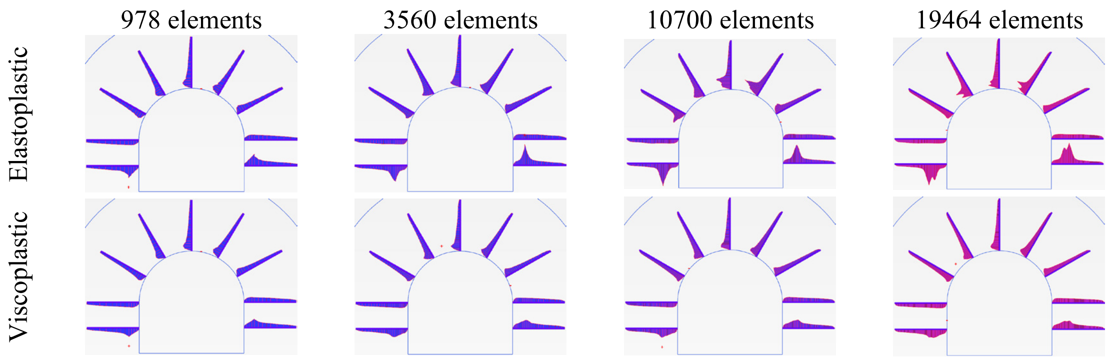


Figure 6. Axial forces developed in the grouted cable bolts with varying mesh sizes.

These aspects are much more remarkable when examining the evolution of maximum axial forces with the increasing number of elements in the mesh (Figure 7). The results, in particular for elements 1, 4, 5, 6 and 9, highlight the strong mesh-dependency of the elastoplastic solution where there is no specific trend of the values with the increasing number of elements but rather randomness of the computed forces. Indeed, if being cut by shear bands, a bolt might be stretched, leading reasonably to a peak value of the normal force (see Figure 6). However, when the numerical solution is non-unique due to strain softening or plastic non-associativity, the formation and size of these shear bands are mesh dependent, and so are the bolt forces. For the regularized solution, the numerical results tend to a converged unique solution, showing the mesh-objectivity and providing more stable and reliable numerical results. Note that for several bolts, normal forces from the elastoplastic solution are twice (or more) in magnitude than those resulting from the more reliable regularized solutions.

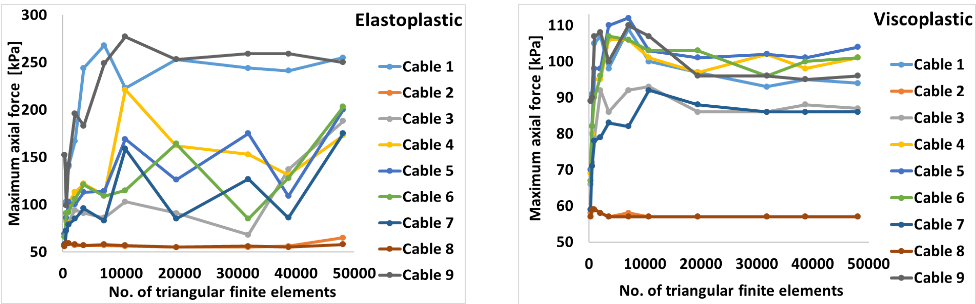


Figure 7. Maximum axial forces developed in the grouted cable bolts with varying mesh sizes.

5 CONCLUSIONS

Advanced numerical techniques have been implemented in the FE code PLAXIS to model bolt-reinforced rock masses exhibiting softening response. The performance of these techniques has been illustrated through numerical analyses of a reinforced mine drift. Results have shown that non-regularized elastoplastic solutions lead to a significant mesh-dependency, especially when analyzing forces developed in the bolt elements cut by shear bands. The solution is proven to be much less mesh-dependent by using viscous regularization, leading to more reliable results. In conclusion, such a technique is viable for mitigating the mesh-dependency of the numerical solution of bolt-reinforced rock mass due to strain localization.

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